

DOOSAN Document Transmittal <u>Upper Trishuli-1 HEP (216MW)</u>					Date 2022-08-10	DT No.: UT1-HEP-DHI-PC-D-0055	SHEET 1 OF 1
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1	RN-0213	AN	UT1-C-010-CVL-DC-42005	0D	Structural Calculation of Diversion Tunnel Lining	13 SHEET	
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						Youngsoo Son Project Manager	
Discipline : Civil						Email: youngsoo.son@doosan.com Doosan Enerbility Co., Ltd.	

UPPER TRISHULI-1 HEP (216MW)

Client	Nepal Water and Energy Development Company Private Limited
Owner's Engineer	Joint Venture of Tractebel Engineering GmbH & Jade Consult

REVIEW NOTE

Contractor	Doosan Enerbility		
Title of the Document	Structural Calculation of Diversion Tunnel Lining and RC-0060		
Document/Drawing No.	UT1-C-010-CVL-DC-42005	Revision	0D
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A: Approved AN: Approved as Noted RR: Returned for Resubmission FI: For Information, only

General comments

- Comments to the document *Appendix-1 Backup Plan in Case of Failure of Diversion Tunnel* will be given with a separate review note.
- The suggestion/recommendation to the drawing *UT1-C-010-CVL-DG-42006_0D Reinforcement drawing of DT lining* will be given with a separate review note. The basis of review is presented in this review note.

- The diversion tunnel is a temporary structure used only during the construction period, so the durability of the Diversion Tunnel is not as critical as for permanent structures. However, due to the high water velocity there is high probability of wearing the lining in a few years or even during one extreme flood event, see as an example figure below.



Example of abrading damage of reinforced concrete due to high flow velocities at Upper Trishuli 3A HPP (source: UT1-C-000-CVL-DD-40003_0A "Design Optimization Proposal-II", figure 2.3-3, page 9)

As the number of Diversion Tunnels has been reduced to one, there is no more a backup existing in case of failure of the Diversion Tunnel.
 The contractor shall present a reasonable method statement regarding evacuation of the effected construction sites and recovering plan of the river diversion facilities in case of failure of the Diversion Tunnel to guarantee safety and quick reaction to reduce the impact of such a failure.

Reply: Please refer attached Appendix-1 documents with this reply comment.

2-1) For this temporary structure, the durability and water tightness is not taken as major issue and the load factors adopted by the contractor are considered satisfactory. (This means, the load factor of EM 2104 is not considered by contractor and only EM 2901 is followed, same has been considered for the review)

Reply: As diversion tunnel is temporary structure the load factor of EM 2104 is not considered and only EM 2901 is followed.

2-2) It is found that rock loads have been neglected as it would be stabilized prior to the lining. Now the remaining loads are self-weight, grout pressure, external and internal water pressure. Though the loading conditions are same except rock spring stiffness) the section size and reinforcements are significantly different even for the same class rock in the contractor's report.

Though the contractor was requested to recheck the spring stiffness calculation, the spring values are still the same as per previous report. So, the consultant has given the clear calculation in the discussion section and the contractor is suggested to compare their calculation with consultant.

Reply: Accept. The spring stiffness are calculated as below table.

Rock Class	modules	b	Poisson ratio	θ	Normal stiffness	Tangential stiffness
	kPa	m		Rad	kN/m	kN/m
II	2.05e7	1.0	0.25	0.087266	1431170	572468
III	7.5e6	1.0	0.27	0.087266	515353	202895
IV	2.30e6	1.0	0.30	0.087266	154395	59383

OE Comment: OK noted.

A modeling for concrete lining is done in SAP2000 for class III rock with following inputs:

- Section size : 500mm
- Grout pressure : 200 kPa
- External water pressure : $3 \times \text{tunnel height} = 18\text{m}$

Following are the results:

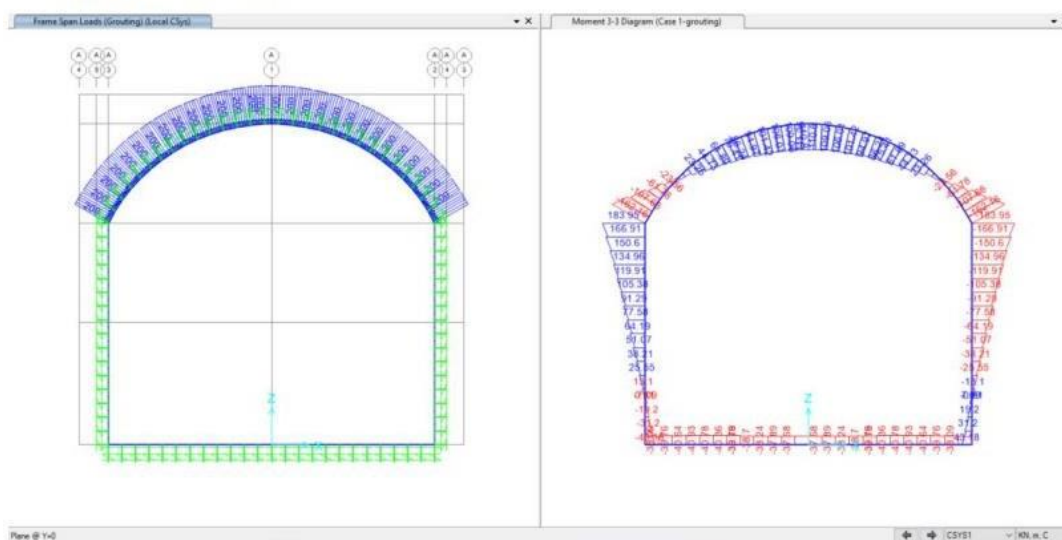


Fig 1: Grout load at the crown only (left), the bending moment diagram for case 1 (right)

Case 1:

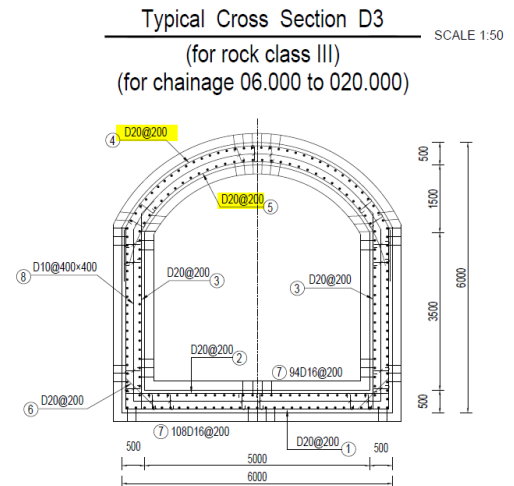
$1.1 \times \text{Self weight} + 1.4 \times \text{Grout load (200 kPa)}$

The moment diagram clearly shows that singly reinforced section is not adequate due to the presence of both positive and negative moments in the section.

Reply: If grouting pressure 200kPa is applied to the whole top arch, the bending moment result of contractor' calculation result ($1.1 \times \text{self-weight} + 1.4 \times \text{grouting pressure}$) is as follows:

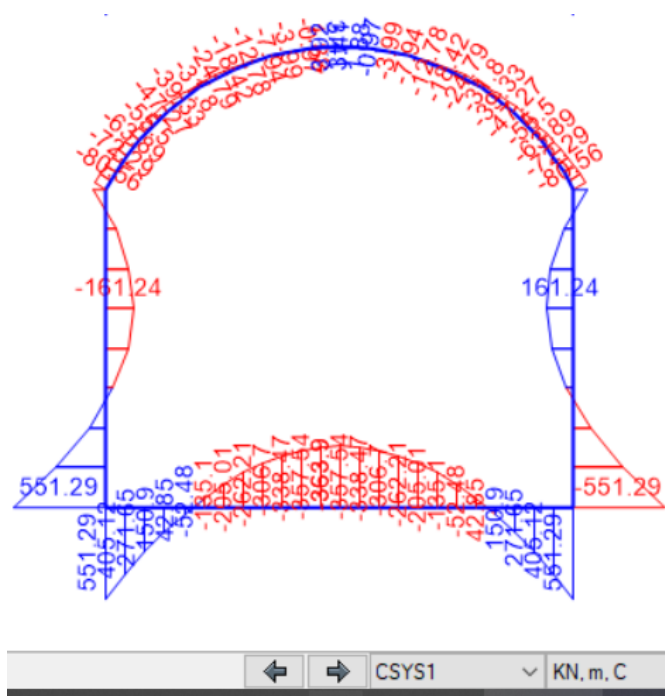
OE Comment: As seen from the moment diagram (by contractor and consultant), the governing moment in the arch portion is case 1. When checked in the reinforcement diagram (UT1-C-010-CVL-DG-42006_0D Reinforcement drawing of DT lining), the top arch reinforcement of D20@200 satisfies the moment output from SAP and MIDAS as well. The adjacent diagram is from the reinforcement drawing.

But for the vertical walls, and other classes (II and IV) the suggestions on the reinforcement quantities have been made as per the consultant's output based on other cases. The suggestions could be found in the reinforcement drawing.



For example:

When the external water pressure is considered and the tunnel is emptied, then the moment diagram is as following for class II: ($1.1 \times \text{self-weight} + 1.4 \times \text{external water pressure}$)



The moment diagram shows both positive and negative bending moments demanding doubly reinforced section.

Some discrepancies found in design report compared to the manual calculation:

For a general structure with external fluid pressure, the manual calculation of axial load at the bottom of the arch member is given below:

With simple engineering principal,

The total vertical grouting load on the arch = grout pressure*tunnel width =
 $200 \text{ kPa} \times 5.6 \text{ m} = 1120 \text{ kN/m}$. The load is distributed equally on two sides, so the axial load on each side =
 $1120/2 = 560 \text{ kN/m}$.

The total self-weight load of arch = section thickness * arch length*specific wt of concrete =

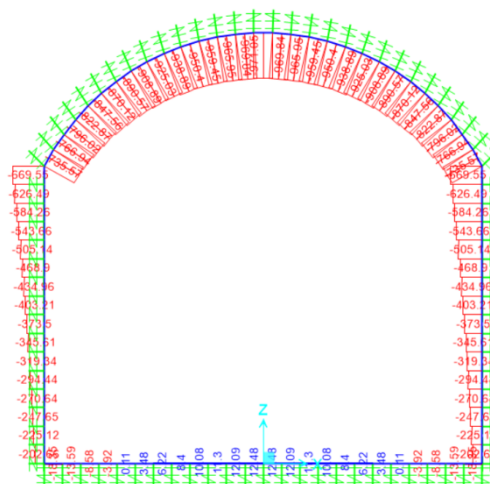
$= 0.5 \text{ m} \times 6.9 \text{ m} \times 25 \text{ kN/m}^3 = 86.25 \text{ kN/m}$,

The load on each side = $86.25/2 = 43.125 \text{ kN/m}$

So, case 1 axial load = $1.1 \times \text{self-weight} + 1.4 \times \text{grout load}$

$= 1.1 \times 43.125 + 1.4 \times 560 = 831.44 \text{ kN/m}$

The SAP analysis gives 735 kN/m when normal stiffness and tangential stiffness are used. The SAP output is given below:



But when the contractor's design report (UT1-C-010-CVL-DC-42005_0D Structural Calculation of DT Lining) Figure 4.1.1-2 is checked the axial forces for axial forces for case 1 (class III rock), the axial force is just 400 kN/m, that is 2 times less than that from manual calculation (and SAP as well). The screenshot of the axial load output from contractor's report is shown below:

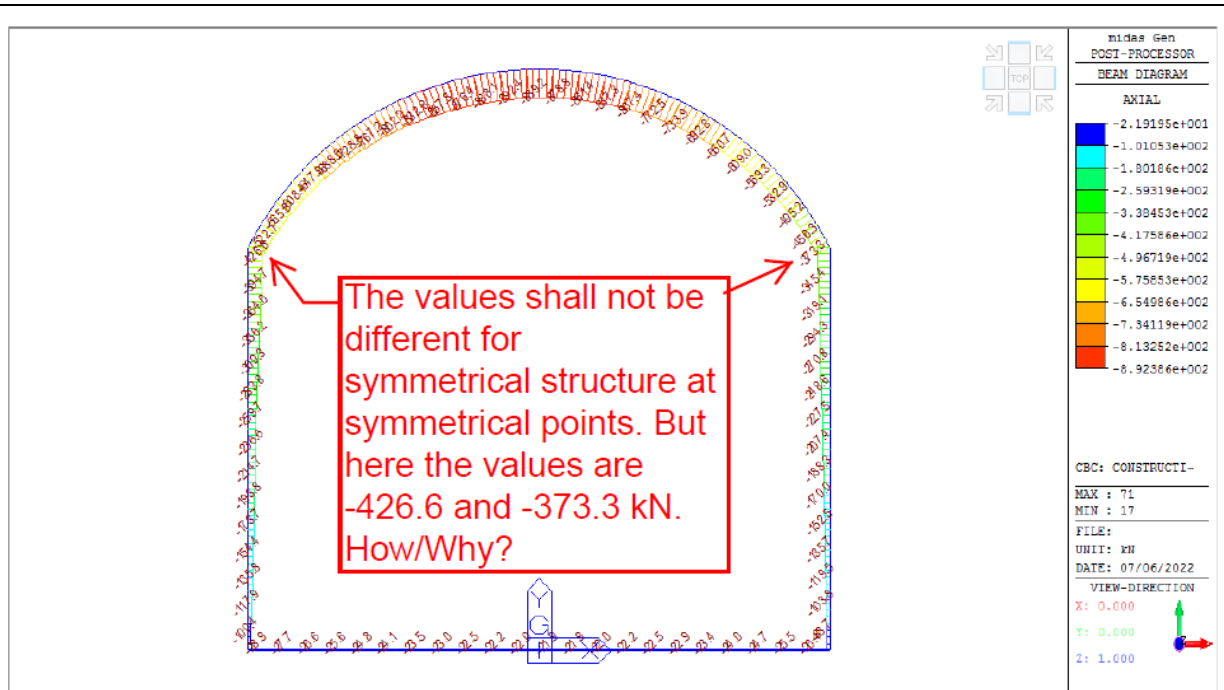


Figure 4.1.1-2 Figure of Axial Force at Concrete Lining Section (CASE1)

So, the contractor could check the some critical result by manual calculation to verify the output. It is to be noted that EM 2901 does not take the effect of rock spring stiffness while calculating the loads from external fluid pressure.

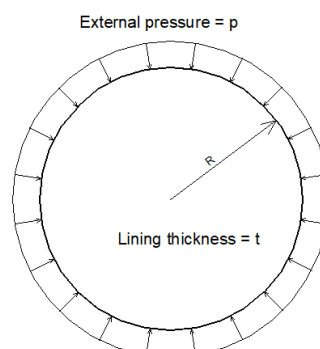
For more clarity, the contractor can also go through axial load calculation of circular due to external pressure as per EM 1110-2-2901, cl 9-4d. The circular tunnel axial load calculation gives the understanding of axial load calculation for arch portion. Since the diversion tunnel contains circular arch, the calculation of circular tunnel axial load is present here as per EM 1110-2-2901, cl 9-4d.

The consultant has elaborated the manual calculation in order to avoid any conflict. The calculation is only for the axial load. The calculation is as following:

External pressure = p

Lining thickness = t

(Axial) Stress in concrete lining = $f_c = pR/t$



Calculation as per EM 2901 Cl, 9-4 d

Lining thickness = t

(Axial) Stress in concrete lining = $f_c = pR/t$

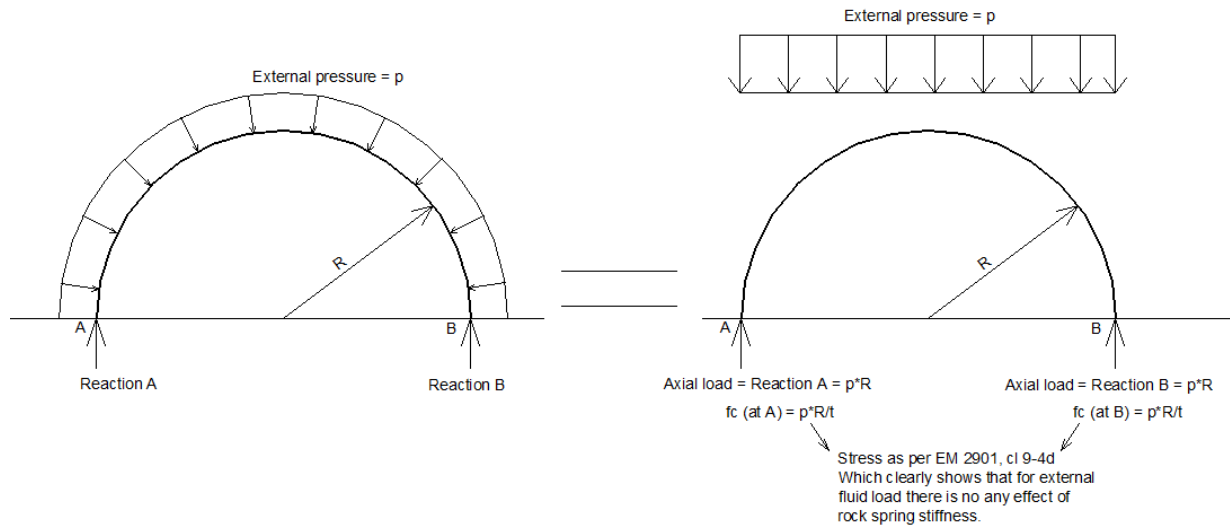
Figure. Axial load calculation for the circular tunnel as per EM 2901 cl 9-4d

The derivation principal of the axial load is given below:

Total vertical load on the arch = $p \cdot 2R$

Load on each arm/side = $p \cdot 2R/2 = pR$

Axial stress on each arm = pR/t



This shows that, for the external fluid pressure, EM 2901 cl 9-4d does not consider the effect of rock spring stiffness. The spring stiffness is activated only when there is compression of the lining against the supporting rock.

In case of empty condition, if the software result varies significantly between spring (stiffness) assigned condition and spring removed condition then the manual verification might be necessary to verify the software results.

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ACI COMMITTEE REPORT

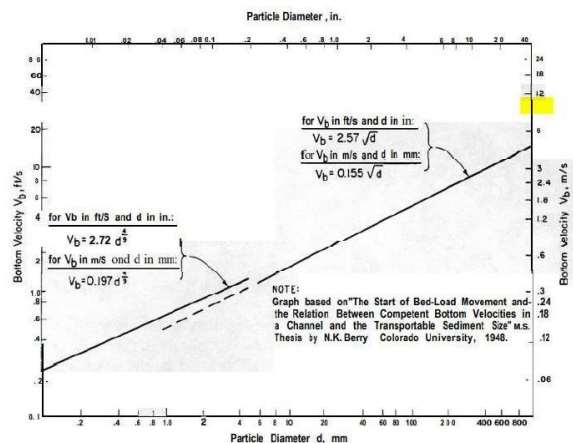


Fig. 3.2-Bottom velocity versus transported sediment size

Plugging Case:

1.1 *Self weight + 1.4*External water pressure (18m=3*tunnel height)

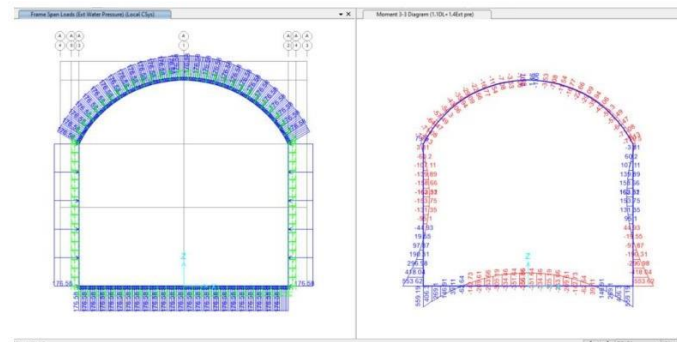


Fig 2: External water pressure (left), the bending moment diagram for plugging (right)

So, the calculation gives at least 25mm dia bars @ 155 mm c/c at the base of vertical wall at the rock side. Similarly, the bottom reinforcement of base slab demands 25mm dia bars @155mm.

But the reinforcement drawing provided by contractor (UT1-C-010-CVL-DG-42006_0C Reinforcement drawing of DT lining) shows the less reinforcement.

Reply: In the plugging case, the external water pressure may be plays a controlling role. But according to construction schedule, the plugging construction may be lasted within 2 months. Meanwhile, the plugging is arranged in March belong to dry season; the water level is lower. The possible external water pressure acting on concrete lining is small. Even if there is a large water pressure, the concrete cracks will not be a safety risk. Once the plugging is completed, the diversion tunnel will lose its function and need no further care. So, plugging case can be removed.

OE Comments:

Since EM 2901 loads (Not ACI 2104) is used, the design does not include the crack check which is considered satisfactory for the temporary structure. So, cracks are not considered as significant issue. But as per the reference to sub-chapter 5 of the report (Appendix-1 Backup plan in case of failure of diversion tunnel), the damage assessment of the diversion tunnel shall be carried out. The screen shot from the report is shown below:

5 Treatment plan in case of failure of the diversion tunnel (If it can be repaired)

According to the diversion scheme of the project, the diversion tunnel plus river channel is used for the dam diversion in monsoon season and the work of dam foundation pit is stopped. After the main monsoon season, the damage assessment of the diversion tunnel should be carried out. If some structural damage occurs in the tunnel, affecting the flow capacity of the diversion tunnel, it can be repaired during dry season as following shown.

So, for the damage assessment the tunnel needs to be emptied after every monsoon. In that case, the governing load would be the external water pressure. But this load case has been eliminated by the contractor in the recent reports though it was considered in the first submission. From the SAP analysis it is seen that the design of the vertical wall and base wall are governed by the load case in the emptied condition. If the lining is not designed for

the governing condition, the probability of tunnel failure is very high during the inspection. Though there might not be risk to life, the highly deformed lining due to the structural failure will not be able to serve the purpose.

In the earlier review notes, the consultant has requested to consider the case for empty tunnel condition including minimum external pressure as per EM 2901, cl 9-1h. So, same requirement is again requested. The load case for empty tunnel condition also needs to be considered for design. The reinforcement provided in present drawing is not enough.

Besides, it is not clear why the different section sizes have been used for same class rock III. Section sizes of 300mm and 500mm is used for rock class III.

Reply: At the beginning of the diversion tunnel design, the lining thickness was designed just based on the Contractor's experiences, at the locations where the overburden rock thickness are small, the lining should have a bigger thickness, vice versa. Therefore, in our design, at locations, for example, inlet area, outlet area and tunnel plug area (considering headrace tunnel intersects overhead nearby), the lining thicknesses are bigger, while in other areas, the lining thickness is smaller. Now it is deemed a little bit conservative.

OE Comment:

Contractor's experience is very useful for successful completion of project but every site is unique and the requirement varies from site to site. The load condition (especially the external water pressure) might not be the same as in other projects. Since the consultant understanding is that, the governing load condition is almost same in all the rock class, the design shall not vary significantly from place to place (except in some critical locations as in intersection, inlet etc).

ACI 318-19, shall be used for design purpose of the project as there are some changes regarding the shear design, hook length etc. The changes are as per the latest research. Mixing ACI 318-14 will create confusion. In case the contractor's software does not include ACI 318-19 but just include ACI 318-14, then those items which are changed in ACI 318-19 (compared to ACI 318-14) need to be verified manually.

Reply: Confirmed to apply ACI 318-19.

2-1) The concrete compressive strength adopted for lining is 24MPa in section 3.1 (1). a. of the report, but ACI 308-06, clause 1.1.1 requires the minimum compressive strength of 28 MPa for hydraulic structures. Besides, the flow velocity is not considered in design of lining strength. As per the discussion with geotechnical expert, it is found that the flow velocity is greater than 10m/s. As per ACI 308-06, fig. 3.2, it is seen that 10m/s bottom velocity could transport the particle size of around 800mm and greater. The sediment size might not be so big in our case but there is no any calculation regarding the abrasion of concrete cover thickness. The fig. 3.2 from ACI 308-06 is shown below:

2-2) The cover dimension mentioned in the section diagram is unclear and seems less than 100mm. Please mention the clear cover adopted for the design of concrete lining in the design report.

Reply:

Firstly, regarding applicable standards, as per hydroelectric project engineering practice, the diversion tunnel is a temporary hydraulic structure, contractor deems its the design should follow EM 2104 and EM 2901 instead of ACI 350, since ACI 350 is mainly related to

environment concrete structure requiring high liquid-tightness and gas-tightness such as tank, reservoir.

Secondly, the concrete strength and concrete cover should be determined by considering of the flow velocity and bedload condition, but they are difficult to be determined by calculation.

For concrete strength, as per Employer's Requirement, it is clearly specified that the concrete strength of diversion tunnel is 24MPa. by considering the flow condition of diversion tunnel, contractor himself would like to increase the invert slab concrete strength to 30MPa at his cost, and maintain 24MPa for sidewall and roof lining concrete unchanged.

For concrete cover of invert slab, it can be increased to 150mm.

OE Comments : The use of concrete strength of 30 MPa for the invert of the Diversion Tunnel lining is acceptable. However, the comments under general comments, subchapter 1 of this review note shall be adopted.

Reply: Noted.

2-3) An article titled "Field Investigation on Hydro-abrasion in High-Speed Sediment-Laden Flows at Sediment Bypass Tunnels" by Michelle Muller-Hagmann, Ismail Albayrak, Christian Auel and Robert M. Boes, 2020 suggest that for 9.8m/s velocity the average depth of wearing for M110 concrete is around 31mm in operational four years. The maximum wearing depth is roughly twice the average wearing depth, that is, about 62mm as per the paper.

Reply: when the flow velocity inside the tunnel and bedload condition as dam site are considered, 150mm concrete cover for invert slab will be enough.

2-4) This suggests the need to high strength concrete lining the tunnel. Some alternative could be high strength material like Granite layer lining, steel lining etc.

Reply: Please refer to reply to 2-2) above.

3) Round bar of 280 MPa is not available in Nepal. Generally, 415 MPa or 500 MPa bars are used as shear bars. So, please use the available bars in the design.

Reply: Accepted, 500Mpa bar will be used.

4) Table 3.2-1, shows the normal and tangential spring stiffness of surrounding rock (Unit length). The beam lengths are divided into arc length corresponding to 5-degree central angle, the spring constant for each beam element needs to be calculated based on that length. However, the provided spring stiffness is calculated for 1m length and is about 4 times greater than that required for 5 degree arc length. In addition, the discretization length of the tunnel at the vertical face and invert level are different as seen from the figure 3.2-2. Please provide the calculation for spring constant for those parts as per the length of the element.

Reply: in our calculation sheet, the beam length arc is 5 degree, and unit length or 1m should be adopted along tunnel. Please refer to ASCE Vol 2-Civil Engineering Guidelines for Planning and Designing Hydroelectric Developments-Waterways, page 3-47 and 3-48.

OE Comments:

The spring stiffness values in the contractor's report are 4 times greater than the calculation as per EM 2901 formula (9-18). The contractor is requested to verify their calculation with consultant. The screenshot of the calculation is given below:

UT-1: Calculation from Consultant	
EM 1110-2-2901, formula (9-18)	
Spring Constant Calculation for Rock Class II	
Er =	20500 MPa
Mean tunnel radius, r =	2.983 m
Poisson's ratio, v =	0.25
Calculation:	
Circumference, C =	18.74 m
Angle subtended by each arc element, θ =	0.08727 radians
Length of element considered, b (flow dirtn) =	1.00 m
$kr = Er \cdot b \cdot \theta / (1+v) =$	1431170 kN/m
$= 20500 \cdot 1000 \cdot 0.08727 / (1+0.25) =$	1431170 kN/m
$kt = 0.5 \cdot kr / (1+v) =$	572468 kN/m
$= 0.5 \cdot 1431170 / (1+0.25) =$	572468 kN/m
As per the contractor's report	
kr =	5857143 kN/m
kt =	2342857 kN/m

Reply: Please refer to the reply to the general comments 2-2).

5) Table 3.3-1 considers the grout pressure on the design but the value adopted for the grout pressure is not mentioned in the report. Please provide the grout pressure considered for the design.

Reply: Accepted. The backfill grouting pressure is 0.2MPa, using maximum permitted grout pressure over area equal to one-quarter the tunnel diameter, as per item 8 in Box 9-1 of EM 2901.

OE Comments: ok.

Reply: Noted.

6) If excavation is to be carried out by explosive then horizontal force needs to be increased by 30% as per EM 1110-2-2901, Box 9-1. But 3.3.2 (1) does not consider extra 30% load. If the explosives are not to be used, please mention the method of excavation.

Reply: As per EM 2901, page 9-7, item f, the rock is stabilized at the time the concrete lining is placed, and there are four circumstances when other loads are considered. To the diversion tunnel of this project, none of them is applicable, so the rock load will be not considered in the calculation, and this calculation will be revised without considering rock load.

f. Linings subject to bending and distortion. In most cases, the rock is stabilized at the time the concrete lining is placed, and the lining will accept loads only from water pressure (internal, external, or both). However, reinforced concrete linings may be required to be designed for circumferential bending in order to minimize cracking and avoid excessive distortions. Box 9-1 shows some general recommendations for selection of loads for design. Conditions causing circumferential bending in linings are as follows:

- Uneven support caused a thick layer of rock of much lower modulus than the surrounding rock, or a void left behind the lining.
- Uneven loading caused by a volume of rock loosened after construction, or a localized water pressure trapped in a void behind the lining.
- Displacements from uneven swelling or squeezing rock.
- Construction loads, such as from nonuniform grout pressures.

OE Comments: For the stabilized rocks it is satisfactory. [Reply: Noted.](#)

7) As per EM 1110-2-2901, cl 9-1h, with proper drainage, the external water pressure shall not be taken less than lesser of 25% of the existing external water pressure and three tunnel diameter due to the possibility of clogging. The value adopted for external pressures seem to be very low as per the report. Besides, the minimum external water pressure as per EM 1110-2-2901 cl 9-1h is 18m in our case.

As per EM 1110-2-2901, cl 9-4d it is reasonable to consider external water pressure equal to internal water pressure. The internal water pressure ranges from 16.5m to 28.5m. Hence, the contractor is recommended to adopt the external water pressure value of higher of 18m and internal pressure at the section. So please revisit the calculation.

Reply: Comments accepted.

After careful analysis, contractor consider that the control condition of lining is construction case. The reason is as follows:

In the plugging case, the external water pressure may be plays a controlling role. According to construction schedule, the plugging construction may be lasted within 2 months. Meanwhile, the plugging is arranged in March belong to dry season; the water level is lower. The possible external water pressure acting on concrete lining is small. Even if there is a large water pressure, the concrete cracks will not be a safety risk. Once the plugging is completed, the diversion tunnel will lost its function and need no further care.

OE Comments:

- The load case for plugging (1.1 self weight + 1.4 external water pressure) is eliminated in the updated report. The screen shot from contractor's report is given below:

Table 3.3-1 Operation Conditions and Relevant Loads

No.	Working Condition of Calculation	Self-weight	Internal water Pressure	External water Pressure	Grouting pressure
CASE1	Construction of D/T	√			√
CASE2	Normal operation	√	√	√	

Note:

1. For lining and segment lining with weep holes, the internal pressure equals to the external pressure, thus the water pressure acting on the lining takes zero. However, for lining and segment lining without weep holes, external water pressure is not considered.

2. As for the lining without weep holes, the counteracting effect of external water pressure on internal water pressure is not considered. The force acting on the lining adopts the whole internal water pressure.

However, in general it is the governing case and generally shall not be eliminated as per EM 2901, section 9-1h. The related screen shot from EM 2901 is given below:

instituted to reduce the flow. The lining should be designed to withstand a proportion of the total external water pressure because the drains cannot reduce the pressures to zero, and there is always a chance that some drains will clog. With proper drainage, the design water pressure may be taken as the lesser of 25 percent of the full pressure and a pressure equivalent to a column of water three tunnel diameters high. For construction conditions, a lower design pressure can be chosen.

If in case the construction schedule varies or if the ground water table is higher than expected, than eliminating external water pressure would be problematic.

[Reply: Please refer to above reply.](#)

OE Comments:

Please refer to above reply.

8) The roof water reduction for intersection provided in table 3.7 is not matching with figure 3.3-3. Please revisit the calculation.

Reply: the external pressure calculation method is changed to OE's commented method, the calculation has been revised.

9) The crack width calculation for the lining is not provided in the report. In order to avoid the crack check the load combination shall be multiplied by environmental durability factor (Sd) as per ACI 350-06, clause C.9.2.9.

PC Reply: Please refer to reply to 2-2) above.

OE Comments: OK

Reply: Noted.

10) The minimum reinforcement A_s , min shall fulfill the requirement of ACI 350-06, Table 7.12.2.1. Though the reinforcement will be checked only after incorporating the comments, it is seen that the reinforcement provided does not meet the requirement of minimum requirement in some places.

Reply: EM 2104 is a specialized standard for hydraulic structures, so we will follow EM 2104 in determination of shrinkage reinforcement. Contraction joints will be applied with a space of 6.3m, and the minimum shrinkage reinforcement ratio takes 0.003.

OE Comments: The EM 2104 and ACI 350-06 agree with each other. Though the load factors adopted for this structure is not as per EM 2104, it is considered satisfactory as the governing aspect will be rapid

wearing of concrete rather than durability. If contractor assures that the work will continue even if diversion tunnel stops working then the minimum reinforcement is not an issue.

Reply: Please refer to attached Appendix-1 document with this reply comment.

11) Please provide the software files (Phase 2D and Midas Gen files) used in this report, so that it could be verified quickly.

Reply: The software files (Phase 2D and Midas Gen files) used in this report is attached in the mail for quickly verifying purpose.

OE Comments: OK.

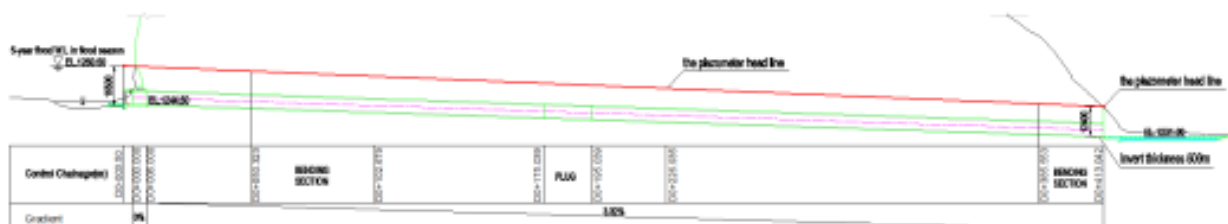
Reply: Noted.

OE additional comments:

Some changes observed in the new report- The updated report has adopted the internal water pressure as 16m only where as it was 28.5m in the earlier report. What is the reason beyond that?

Reply: Please refer to subchapter 3.3.2 Calculation of Loads. The internal pressure within tunnel under normal operation case from inlet to outlet ranges from 16.5m to 12.4m, for safety consideration, 16.5m is selected as the internal pressure. 16m in the Rev.0C is a mistake.

Internal pressure of 28.5m in earlier revision is improper for without considering head-loss and velocity head. The piezometer head line is as follows:



OE Comments: OK

DISCLAIMER

Any approval will not release the Contractor from any of his obligations under the Contract. Despite the Engineer's approval the Contractor remains fully responsible for design, calculation, dimensioning, supply and installation without limitations.



Dr. Subin Desai (Structural Engineer)



Sebastian Beck (Chief Design Engineer)